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Shear strength of glued laminated timber made from European beech timber

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Abstract This paper describes an investigation into the shear strength of glued laminated timber (GLT) made from European beech. Special consideration was paid to the possible strength influence of a frequently occurring discolouration of the timber, termed red heartwood, which is inherent to the species. The beech lamellae were visually graded according to German hardwood grading standard DIN 4074-5. Grade LS13, conforming to European hardwood strength class D40, was split into two sub-grades LS13− and LS13+. Additionally, modulus of elasticity (MOE) was determined by ultrasound pulse, longitudinal vibration and static tension tests. Sub-grade LS13+ showed a mean density and MOE of 690 kg/m^3 and 14,800 N/mm^2, respectively.

The GLT shear strength was evaluated by means of four-point bending tests on structural sized I-shaped beams with a depth of 0.6 m and a span to depth ratio of 5:1. The slightly inhomogeneous build-up of the cross-section conformed to glulam strength class GL42c. Two beam samples were investigated, each with seven specimens, where one grouping had no red heartwood and the other with a high red heartwood in the web laminations. Additionally block shear tests on bond line strength were performed with standardized small specimens according to EN 392.

Neither the beam shear capacity tests nor the bond line block shear tests revealed an influence of the red heartwood discolouration on strength. The fifth-percentile value of shear strength of all beams was 3.5 N/mm^2. The results of the block shear tests indicate that the present requirements on minimum block shear strength are set too low in the European standard EN 386 with regard to beech GLT.

Schubfestigkeit von Brettschichtholz aus europäischem Buchenholz

Zusammenfassung Der Aufsatz berichtet über Untersuchungen zur Schubfestigkeit von Brettschichtholz (BSH) aus Europäischer Buche. Besondere Beachtung wurde dem eventuellen Festigkeitseinfluss einer häufig auftretenden holzartspezifischen Verfärbung des Holzes, Rotkern genannt, zugemessen. Die Buchenholz-Brettlamellen wurden visuell nach der deutschen Laubholzsortiernorm DIN 4074-5 sortiert. Die Sortierklasse LS13, die der Europäischen Laubholzfestigkeitsklasse D40 zugeordnet ist, wurde in die Klassen LS13− und LS13+ untergliedert. Zusätzlich wurde der Elastizitätsmodul mittels Ultraschalllaufzeit, Längsschwingung und statischer Zugversuche bestimmt. Die Sortierunterklasse LS13+ wies eine mittlere Rohdichte von 690 kg/m^3 und einen Elastizitätsmodul von 14.800 N/mm^2 auf.

Die BSH-Schubfestigkeitsuntersuchungen wurden mittels Vier-Punkt-Biegeversuchen an großformatigen I-Trägern mit einer Querschnittshöhe von 0,6 m und einem Stützweiten-Querschnittshöhenverhältnis von 5:1 durchgeführt. Der geringfügig inhomogene Querschnittsaufbau entsprach der BSH-Festigkeitsklasse GL42c. Die Untersuchungen umfassten zwei Trägerkollektive, mit je sieben Prüfkörpern, jeweils ohne bzw. mit hohem Rotkernanteil in den Steglamellen. Weiterhin wurden Blockscherversuche zur Klebfugenfestigkeit an kleinen Normprüfkörpern nach EN 392 durchgeführt.

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Weder die Schubtragfähigkeitsversuche an den Biegeträgern noch die Klebefugen-Blockscherversuche zeigten einen Festigkeitseinfluss der Rotkernverfärbung. Die 5%-Quantile der Schubfestigkeiten aller Träger ergab sich zu 3,5 N/mm^2. Die Ergebnisse der Blockscherversuche weisen darauf hin, dass die derzeitigen Anforderungen an die Klebfugenscherfestigkeit in EN 386 in Bezug auf Buchen-BSH zu niedrig sind.

1 Introduction

The higher strength and stiffness properties of beech wood as compared to most softwood species are well known. The relative ease of glueability of beech wood is highlighted in the proliferation of beech wood objects manufactured in the solid wood furniture industry where beech represents the predominantly used material in Europe. The possibility of using beech timber for structural glulam beams was first successfully demonstrated in the mid-sixties by Egner and Kolb (1966) and Kolb (1968). Later Gehri (1980, 1985) once again pointed out the high strength and stiffness potential of beech glulam. However, up to today, glulam made of beech timber has been used only on rare occasions. This situation should change considerably in the coming decades due to several reasons, inter alia altered silvicultural policies and the need for timber products with enhanced performance characteristics. Within the last ten years a considerable number of investigations focused on several aspects of structural use of beech timber such as grading, finger joint strength, glue line integrity, beam strengths, influences of red heartwood (Glos and Lederer 2000; Aicher et al. 2001; Frühwald et al. 2003; Bernasconi 2004; Pöhler et al. 2006; Blaß et al. 2005; Frese and Blaß 2005; Frese 2006; Aicher and Reinhardt 2007; Ohnesorge et al. 2006, 2008). The most comprehensive investigations on grading of beech timber, strength of beech lamellae as well as bending strength and stiffness of beech glulam beams were performed by Glos and Lederer (2000), Blaß et al. (2005) and Frese (2006). The latter investigations laid the foundations for introducing beech glulam as a designable and reliable building product in the construction market by means of a recently granted general building approval Z-9.1-679 (DIBt 2009). For the time being, the use of beech glulam is restricted to service class 1 as defined in EN 1995-1-1/A1 (2004/2008) due to low natural durability and relative high shrinkage/swelling coefficients of the timber.

Despite these promising steps, some basic questions related to beech glulam remain to be resolved. One major issue is (bond) shear strength, which becomes the most important structural material property, next to bending strength, in the design of glulam beams in certain cases of load configurations and span to depth ratios. At present, the specification of characteristic shear strength of beech glulam is not based on consistent investigations on this material property relevant for structural sized beams. Investigations on this issue are presented in this paper.

2 Material and test specimens

The material for this investigation was purchased in 2005 from a hardwood sawmill in the state of Baden-Württemberg, Germany. A total of 28 logs, already sawn into boards of 45 mm thickness, were specifically selected with regard to the amount of red heartwood that each contained. The boards had been air-seasoned for up to five years under protected external conditions. The red heartwood amount of the logs varied from 0 to 60% of the diameter at the bottom log and the quality grades were B or B/C—according to the former German round wood grading act HKS (1969).

2.1 Beech lamellae grading and material characteristics

Prior to sawing the lamellae to a width of 130 mm, the boards were kiln-dried to an equilibrium moisture content of about 11%. As part of the lamellae sawing process, larger defects such as knots with diameters of more than 8 cm, pith, and transverse cracks were eliminated by crosscutting.

2.1.1 Visual grades, density and red heartwood

538 lamellae with lengths between 1.1 and 4.9 m were visually graded according to the German hardwood grading standard DIN 4074-5 (2003) into the three provided grades LS7, LS10 and LS13—primarily differing with respect to knot ratio. In addition, the highest grade LS13, denoted by knot ratios of 0.2 for single knots and 0.33 for knot groupings, was subdivided into two subclasses LS13− and LS13+ as first introduced by Blaß et al. (2005). Subclass LS13+ is characterized by all features acceptable for grade LS13 except for absolute single knot size, which is restricted to a maximum value of 5 mm. The most important grading and material characteristics are depicted in Fig. 1a–c.

The density of all lamellae was determined from small defect free end slabs according to ISO 3131 (1975). Hereby the individual moisture content was measured following the procedures of EN 13183-1 (2002). The mean moisture content (±SD = standard deviation) of all lamellae was 11.3 ± 1.0%. The obtained frequency distribution of normal density ρ_{12} is depicted in Fig. 1c. The mean density (±SD) was 690 ± 55 kg/m^3; no lamellae showed a density less than 600 kg/m^3.

The red heartwood amount was measured on all lamellae at the edges and wide faces following the provisions of chap. 5.8 of DIN 4074-5 (2003). In total, 51% of all lamellae contained less than 5% red heartwood. The amount of

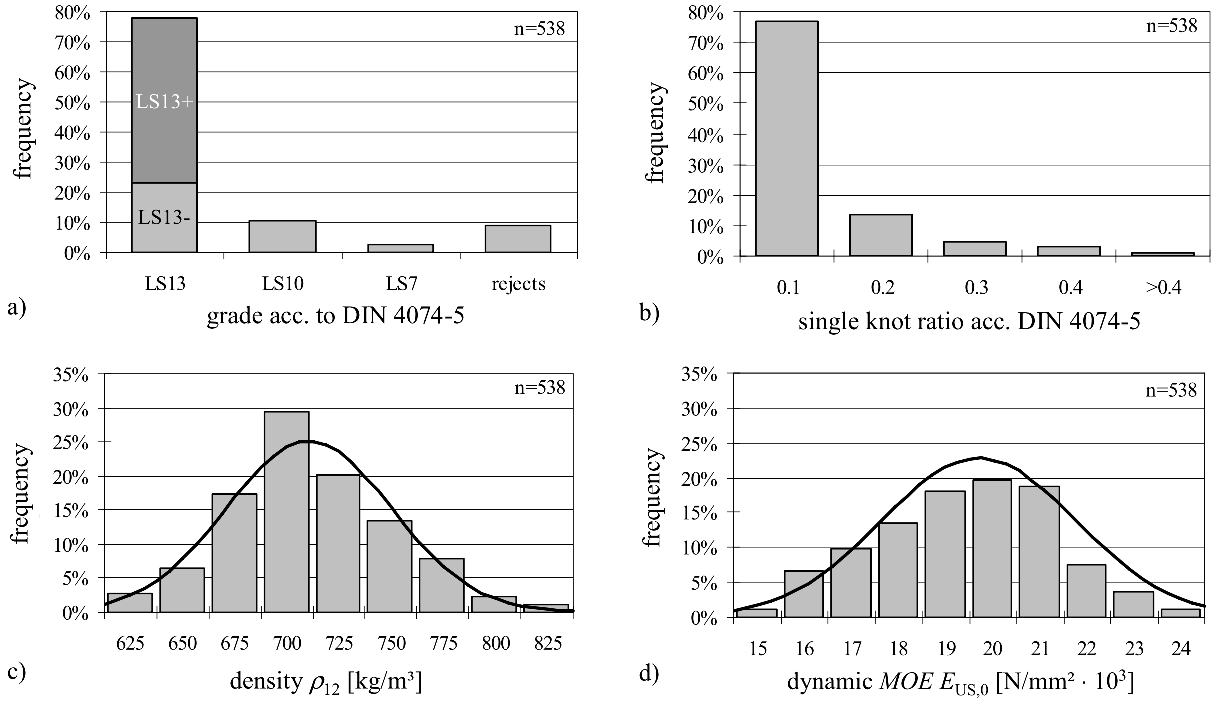


Fig. 1 Grading and material characteristics of all 538 beech lamellae: (**a**) frequency distribution of visual grades acc. to DIN 4074-5 (2003) including subclasses of LS13 acc. to Blaß et al. (2005); (**b**) frequency distribution of single knot ratio acc. to DIN 4074-5 (2003); (**c**) frequency distribution of normal density (ρ_{12}); (**d**) frequency distribution of dynamic MOE ($E_{US,0}$) determined with SylvatestDuo®

Abb. 1 Sortier- und Materialeigenschaften aller 538 Buchenlamellen: (**a**) Häufigkeitsverteilung der visuellen Sortierklassen nach DIN 4074-5 (2003) einschließlich der LS13-Unterklassen nach Blaß et al. (2005); (**b**) Häufigkeitsverteilung des Merkmals Einzellast nach DIN 4074-5 (2003); (**c**) Häufigkeitsverteilung der Normal-Rohdichte (ρ_{12}); (**d**) Häufigkeitsverteilung des dynamischen Elastizitätsmoduls ($E_{US,0}$) ermittelt mit SylvatestDuo®

red heartwood in the discoloured lamellae (those with more than 5% red heartwood) varied roughly from 5% to 70% and was on average 25%, very similar for the edges and faces.

2.1.2 Modulus of elasticity, tension strength

Modulus of elasticity (MOE) parallel to grain direction (E_0) of the lamellae was determined via three different methods: ultrasound (US) pulse velocity ($E_{US,0}$), longitudinal vibration ($E_{LV,0}$) and displacement/strain measurement in uniaxial static tension tests ($E_{t,0}$). The results of the tests are given in Table 1. The US pulse velocity tests were performed with all 538 lamellae. The MOE was determined with regard to—moisture corrected—density influence as for isotropic, homogeneous mediums ($E_{US,0} = v^2 \cdot \rho_{12}$). Figure 1d depicts the respective frequency distribution and Fig. 2 shows the relationship (graph a) between $E_{US,0}$ and ρ_{12} and the $E_{US,0}$ regression equation for the sub-grade LS13+ sample.

Based on several MOE evaluations in the known literature (Ilic 2003; Frese and Blaß 2005), it was assumed and finally shown that the presented $E_{US,0}$ values are considerably higher in comparison to other non-destructive testing (NDT) and direct measurement methods. In order to verify/correct the $E_{US,0}$ values, static uniaxial, longitudinal tension tests with displacement/strain measurements were carried out with 51 lamellae of sub-grade LS13+. The specimens had a test length l of 400 mm ($= 3.3 \cdot b_{lam}$, where b_{lam} = lamellae width) clear of the machine grips. The displacement/strain measurement was performed in the centre of l over a distance of 200 mm following the principles of EN 408 (2003). All tests were done in stroke control with a constant displacement rate of the actuator of 0.04 mm/sec.

The $E_{t,0}$ results given in Table 1 and Fig. 2 conform closely to data from the relevant literature (Frese and Blaß 2005). The mean (±SD) and the fifth percentile of the tension strength $f_{t,0}$ and the relationship between tension strength and modulus of elasticity $E_{t,0}$ (in N/mm^2) obtained for the LS13+ laminations were:

$$f_{t,0} = 92.4 \pm 27.1\ \text{N/mm}^2; \qquad f_{t,05} = 49.2\ \text{N/mm}^2 \quad \text{and}$$
$$f_{t,0} = 0.0107 \cdot E_{t,0} - 67.09\ \text{N/mm}^2; \qquad r = 0.64$$

On average the ratio of MOEs determined by US pulse velocity measurement and direct tension tests for sub-grade LS13+ was $E_{US,0}/E_{t,0} = 1.27$. At present, no fully convincing answers can be given with regard to the reason of the rather large discrepancy of the specific NDT and the direct MOE measurement. In addition to ultra-sound pulse veloc-

Table 1 Modulus of elasticity parallel to grain direction of beech lamellae determined by different test methods
Tab. 1 Elastizitätsmodul parallel zur Faserrichtung von Buchenholzlamellen ermittelt mit unterschiedlichen Prüfmethoden

	Method of MOE determination	No. of specimens	Grade acc. to DIN 4074-5[a]	Lamination thickness t [mm]	Density ρ_{12} [kg/m^3]	Modulus of elasticity E_0		
						Mean [N/mm^2]	SD [N/mm^2]	CV [%]
$E_{US,0}$	Ultra-sound pulse velocity (SylvatestDuo®)[b]	294	LS13+	45	697	18,827	1,919	10.2%
$E_{LV,0}$	Longitudinal vibration (Grindosonic)[c]	32	LS13+	32/36	713	15,283	1,855	12.1%
$E_{t,0}$	Static tension test	51	LS13+	38	690	14,868	2,237	15.0%

[a]LS13 divided in subclasses LS13− and LS13+; see Sect. 2.1.1

[b]SylvatestDuo® from CBT—Concept Bois Technology—frequency 22 kHz

[c]Grindosonic from J.W. Lemmens N.V., Belgium

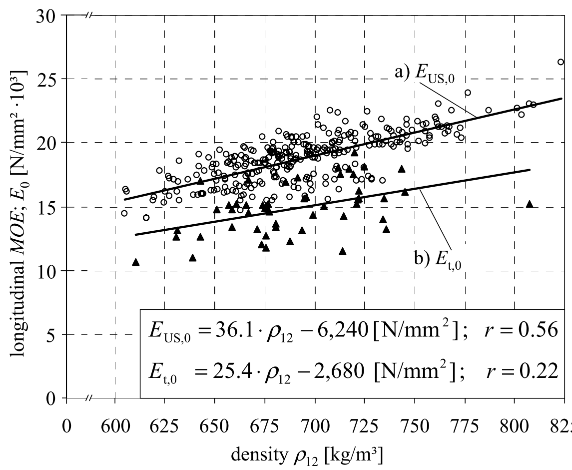


Fig. 2 Relationship of normal density and longitudinal MOE of sub-grade LS13+: (**a**) MOE $E_{US,0}$ based on ultrasonic pulse velocity (○) and (**b**) MOE $E_{t,0}$ based on static tension tests (▲)
Abb. 2 Zusammenhang von Normal-Rohdichte und Elastizitätsmodul parallel zur Faserrichtung der Sortierunterklasse LS13+: (**a**) Elastizitätsmodul $E_{US,0}$ basierend auf Ultraschall-Pulsgeschwindigkeit (○) und (**b**) Elastizitätsmodul $E_{t,0}$ basierend auf statischen Zugprüfungen (▲)

ity and static tension tests, MOE was determined on 32 randomly selected lamellae ($t = 32$ and 36 mm) of sub-grade LS13+ using commercially available longitudinal vibration test equipment. The results given in Table 1, were slightly (3%) higher than the MOEs obtained in the static tension tests.

2.2 I-shaped beam specimens

2.2.1 General comments

Shear strength of glulam in structural sizes cannot be derived directly from small (clear) block shear specimens as defined in EN 392 (1995) or ASTM D143-94 (2007) neither from medium sized specimens according to EN 408 (2003). Small and medium sized specimens may be used, provided apt calibration/adjustment procedures for extrapolation to large scale components exist. Such provisions for instance are given in ASTM D3737 (2008), addressing ASTM D2915 (2003) and ASTM D2555 (2006), to derive allowable glulam shear stresses. Nevertheless, it has been demonstrated (Yeh and Williamson 2001) that the specific approach leads to estimates that are too conservative when compared to results derived from full-scale tests (see also ASTM D3737 2008). In the last three decades different cross-sectional shapes, build-ups and loading configurations have been investigated for determination of shear strength from structural sized glulam specimens (Keenan et al. 1985; Soltis and Rammer 1994; Schickhofer and Obermayr 1998; Yeh and Williamson 2001; Schickhofer 2001). So far, primarily rectangular cross-sections were used. Their use was undertaken partly in conjunction with special provisions that increased the strength in the bending tension zone in order to avoid bending failure. Finger jointed laminations were used by Soltis and Rammer (1994) whereas the specimens by Yeh and Williamson (2001) had no end joints in the tension zone. Keenan et al. (1985) used outer lamellae made of hardwood. I-shaped glulam cross-sections, partly in combination with special outer lamellae (i.e., LVL), were used by Schickhofer and Obermayr (1998) and Schickhofer (2001), hereby following the ideas of Korin's tests (1996) with solid wood. A review of specimen build-ups in the known literature showed that the highest shear failure ratio up to about 100% is obtained by means of an I-shaped cross-section with high strength lamellae without end joints in the bending tension zone. Further, in the given case, no special provisions for preventing crushing on the outer lamination in the bearing areas should be necessary, due to the relatively high

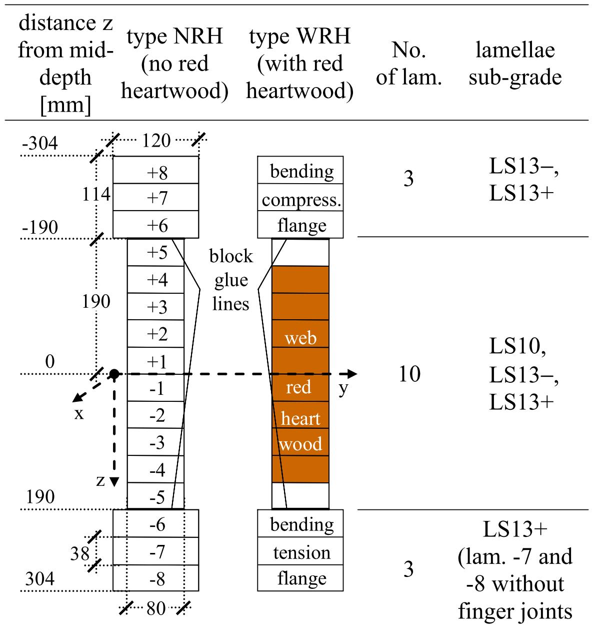


Fig. 3 Cross-sectional build-up, dimensions and numbering of the lamellae for the tested beam types NRH (no red heartwood) and WRH (with red heartwood in the web)
Abb. 3 Querschnittsaufbau, Abmessungen und Lamellennummerierung der Trägertypen NRH (ohne Rotkern) und WRH (mit Rotkern im Steg)

compression strength perpendicular to grain of beech wood in comparison to softwoods.

2.2.2 *Build-up and dimensions*

Figure 3 shows the cross-sectional design and dimensions of the employed beam specimens with a symmetrical I-shaped cross-section. The total depth, width of the web, and beam length were 608 mm, 80 mm and 3,530 mm, respectively. All lamellae had a thickness of $t = 38$ mm. The specimens were first manufactured to a length of 3,900 mm. From one end of the beam with a distance of 100 mm thereof, two cross-sectional slabs were cut with thicknesses of 50 mm for use in block shear and 75 mm for use in delamination tests (not presented in this paper).

The tension flange was constructed entirely of LS13+ lamellae with no finger joint in the outermost lamella. The compression flange was built up predominantly from LS13+ and a few LS13− lamellae, whereas the web consisted of LS13+, LS13− and about 15% LS10 lamellae. The lamellae of the web and of the bending compression flange contained randomly distributed finger joints.

For verification of the potential influence of the amount of red heartwood on shear strength of the timber and on bond shear strength of the glue lines, two specimen types/subgroups, termed WRH and NRH were manufactured, differing considerably by the amount of red heartwood of the eight inner lamellae of the web. In case of subgroup NRH, the respective web lamellae contained almost **n**o **r**ed **h**eartwood. In contrast, in subgroup WRH (**w**ith **r**ed **h**eartwood) the lamellae were characterized by a rather high amount of red heartwood, especially at the glued faces of the lamellae. The percentage of red heartwood in the glued interface ranged from about 60 to 95%. In total, seven specimens of each type (WRH and NRH) were produced.

2.2.3 *Manufacture*

The production of the specimens was carried out in a glulam company with experience in the production of GLT made of hardwoods. A total of 340 lamellae with a cross-section of 45 mm × 130 mm, and an average length of 2.6 m, grouped in (sub)grades of LS10 (14%), LS13− (26%) and LS13+ (60%) were finger-jointed to lamellae of 20 m length. The average moisture content at the time of finger jointing and face gluing was 11.1%. Length and pitch of the employed finger joint profile conformed to 15 mm and 3.8 mm. For gluing the finger joints and equally for gluing the faces of the lamellae a widely used two-component melamine urea adhesive (Kauramin 683 with hardener 688) tested and approved according to the requirements of EN 301 (2006) for the purpose of softwood and beech wood gluing was used in intermixed condition with an adhesive to hardener weight ratio of 100:50. After 30 days of curing the finger joints, and immediately before face gluing, all lamellae were planed to a thickness of 38 mm.

The gluing of the I-shaped cross-section was then performed in a two-step process. First, the flanges, and the web, all with same lamination width (130 mm) were glued separately. After one week of curing, the webs were sawn on one side to a width of 90 mm and were then planed on both sides to a final width of 80 mm. Then the “block-gluing” method was used, when adhering the web to the flanges. In all cases, the adhesive spray was applied on both sides, amounting to about 250 g/m^2 per side. The open assembly time was ≤5 minutes and the closed assembly time ranged from 30 to 50 minutes. The pressure applied during the face gluing process was 1.1 N/mm^2, and was held for 18 hours at about 30°C and 60 ± 10% relative humidity. The total curing time of the specimens in sheltered conditions of 20° ± 5°C before testing was 60 days.

3 Test methods and stress distribution analysis

3.1 Beam specimens

3.1.1 *Test set-up and procedure*

The most commonly used loading methods, in general with a span to depth ratio of about 5:1 to 6:1, are either one-span

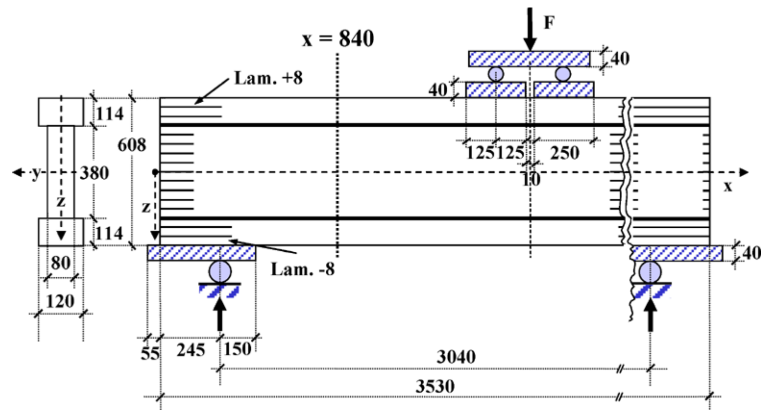


Fig. 4 Test set-up: four point loading
Abb. 4 Versuchsaufbau: Vier-Punkt-Biegeprüfung

tests—some with three-point loading (Schickhofer 2001), some with four-point loading (Yeh and Williamson 2001; ASTM D3737 2008, Annex A 7)—or two-span tests with five-point loading (Soltis and Rammer 1994; Schickhofer 2001). The latter test configuration, first used for advanced composite materials (Jegley and Williams 1988) led to a high shear failure rate of about 70 to 90% in the glulam tests conducted by Soltis and Rammer, which made use of rectangular cross-sections. On the other hand, in shear strength tests with solid wood (Lam et al. 1995) a very low shear failure rate of solely 40% was obtained. Drawbacks to the two-span method are difficulties in detection of crack formation and indications that the obtained shear strengths are higher as compared to one-span test configurations. The latter is most likely due to super-imposed compression stresses perpendicular to grain. Based on the evaluation of appropriate loading methods, a four-point loading configuration with a span to depth ratio of 5:1, shown in Fig. 4, was chosen. The test method is similar to the three-point loading set-up used by Schickhofer (2001). However, concerning the supports, the rollers were not placed centrically under the steel plates (see Fig. 4). Regarding loading, a four-point load situation similar to ASTM D3737 (2008) was applied, enabling better rotation of the specimen. All tests were performed in a servo-hydraulic 1.6 MN test machine in stroke control with a constant rate of piston displacement of 0.1 mm/sec. Thereby ultimate loads were achieved in a range of 142 . . . 231 . . . 375 seconds. The tests were conducted in non-climatised environment at a temperature of about 23°C. The deflection was measured in mid-span by two LVDT's (linear variable differential transformers) mounted at mid-depth on both specimen side faces.

3.1.2 Analysis of stress distributions and deflection

With regard to the evaluation of the test results, a numerical computation of the very compact, plate-like test specimen was performed, due to the fact that its proportions limit the use of beam theory.

In the numerical analysis the specimen and test configuration were approximated using a 2D finite element (FE) model, assuming plane stress conditions. The interfaces between the very stiff loading and support plates were modelled by contact elements. The results given in Figs. 5a–c are based on MOE values $E_0 = E_x = 14{,}500$ N/mm^2 for the flanges and 13,500 N/mm^2 for the web, reflecting the actual build-up of the test specimens. The other stiffness parameters/ratios were chosen as: $E_0/E_{90} = 20$, $E_0/G = 16$, $\nu_{yx} = (E_x/E_y) \cdot \nu_{xy} = 0.4$. The effects of the above stiffness assumptions were checked with parameter studies, which can be summarized as following: small changes of the ratio E_0/E_{90} in the range of about $18 \dots 22$ and a ν_{yx} range of 0.35–0.45 have almost no influence on stresses and deflection. Changes of the E_0/G ratio in the perceivable range of 14 to 18 affect primarily deflection, as the global stiffness of the compact specimen depends significantly on shear modulus.

Figure 5a shows relevant distributions of shear stress along the span at different sections of web depth. A rather constant shear stress field occurs in the middle between the opposite edges of the support and loading plates over a length of about 0.5 m. In this centre part the results of the numerical analysis conform to those from elementary beam theory, especially along web depth (Fig. 5b). With increasing vicinity to the load application plates the FE-shear stress distribution deviates considerably from beam theory as would be expected.

Apart from shear stresses, the compression stresses perpendicular to fibre direction σ_{90} are of major importance, as shear strength will increase in the presence of superimposed compression stresses.

Figure 5c shows how compression stresses perpendicular to fibre vary along beam length at different sections of web depth, i.e., at $z = 0$ (neutral axis) and in the web flange interfaces on the bending tension and bending compression side ($z = \pm 185$ and 190 mm). As anticipated, the compression stresses σ_{90} in the web to flange interfaces decline to zero at some distance from the adjacent support

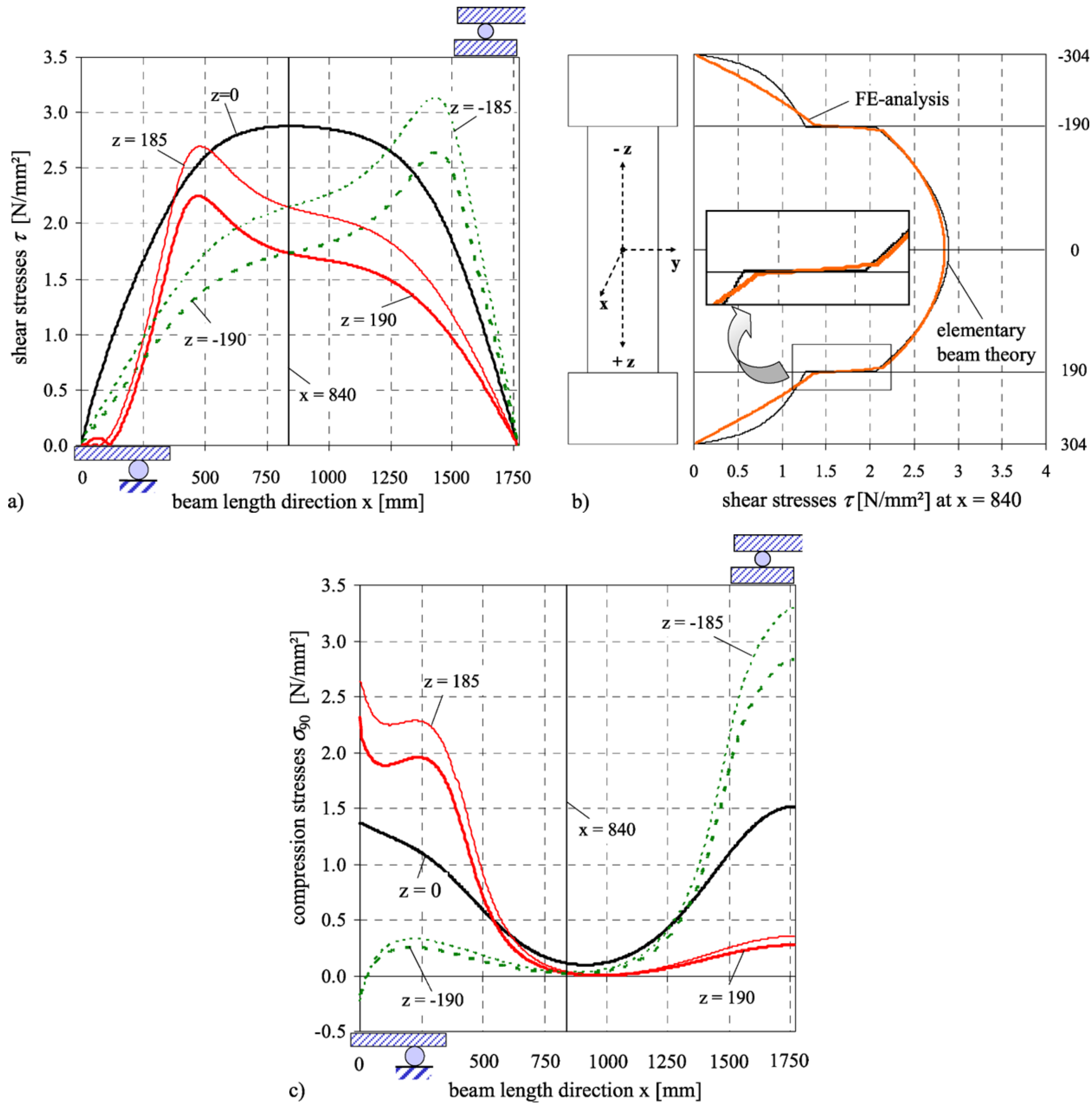


Fig. 5 Stress distributions of the test specimen for a shear force of $V = 100$ kN. (**a**) Shear stresses τ along beam length; (**b**) shear stresses τ at $x = 840$ mm along beam depth; (**c**) compression stresses σ_{90} perpendicular to fibre direction along beam length

Abb. 5 Spannungsverteilungen des Prüfkörpers bei einer Querkraft von $V = 100$ kN. (**a**) Schubspannungen τ entlang der Trägerlängsachse; (**b**) Schubspannungen τ bei $x = 840$ mm entlang der Trägerhöhe; (**c**) Druckspannungen σ_{90} rechtwinklig zur Faserrichtung entlang der Trägerlängsachse

or loading plate. The compression stresses σ_{90} in the neutral axis mirror very roughly the average of the stresses in both web-flange interfaces, however not in the area of rather constant maximum shear stress in the centre part between loading and support plates. Here, compression stresses σ_{90} do not vanish. The non-zero σ_{90} stresses–distributed, parabolically over web depth–result form the inclined compression force, which stretches diagonally from the support plate to the adjacent loading plate in the very compact, plate-like specimen. On average σ_{90} normalized to unit shear force $V = 1$ kN is about 0.002 (N/mm^2/kN). Hence, for mean shear force capacity of all beam tests, being $V_u = 209$ kN (see below), the superimposed compression stress normal to shear fracture plane at mid-depth is about 0.4 N/mm^2. This level of compression stress is rather low and should have little effect on a potential increase of shear strength.

The computational (mid-span) deflection, expressed as mid-span (spring) stiffness $S = F/w\ (\ell/2)$ is 25.77 kN/mm.

3.2 Block shear tests

Block shear tests, used to determine bond line shear strength, were carried out with specimens taken from full cross-sectional slabs with 50 mm thickness, cut from each beam specimen prior to the shear capacity experiments. The tests were performed according to EN 392 (1995) with sticks of 50 mm width cut at mid-width over full depth from the I-shaped cross-sectional slabs. The sticks were conditioned for eight weeks in a climate of 20°C and 65% relative humidity. The testing was conducted with a constant rate of crosshead displacement such that failures occurred within about 20 sec.

4 Results and discussion

4.1 Beam shear capacity tests

The load versus mid-span deflection curves were strictly linear for all specimens up to about 60% of the ultimate shear force capacity $V_u = F_u/2$. Fracture and ultimate load occurred in all cases in a brittle, instantaneous manner, whereby 13 of the 14 beams failed in shear with a crack over the full width of the web. One beam failed in bending due to a bending tension fracture in the outermost lamella. The hereby obtained shear force level conformed to the average of the other beams that failed in shear and hence was included in the evaluation. The lengths of the shear cracks ranged roughly from half to full beam length. The position of the shear crack plane within web depth varied considerably.

The shear failures occurred for six of the 14 beams predominantly within the interface/bond line between two lamellae (Fig. 6). For the other eight beams, a wood shear failure was observed predominantly within the lamellae (Fig. 7). Different to softwoods, the wood shear failures did not follow the growth ring line or the transition of early and latewood. This is due to the diffuse-porous structure of the wood tissue of beech timber.

Table 2 contains a compilation of the most relevant results given separately for the test subgroups WRH and NRH as well as for the combined sample WRH + NRH. The mode and location of failure, average beam density and ultimate shear force capacity constitute the primary test results. Further, the derived shear strength values f_v are given, hereby evaluated for mid-depth of the beam ($f_{v,0}$ for $z = 0$) and for the actual depth location of the fracture plane ($f_{v,f}$).

Throughout the maximum shear stress value in the section $x = \text{const} = 840$ mm was taken. The strength evaluations are presented for both analytical beam theory and FE analysis (see also Fig. 8a and b). The mean and lower fifth percentile value of shear force capacity V_u of sub-sample WRH were about 13% higher as compared to the results of the NRH sub-sample. As the difference in load capacities is statistically not significant, in the following exclusively the results of the combined sample WRH + NRH (14 beams) are discussed.

The mean and fifth percentile values of shear strength $f_{v,0}$ based on maximum shear stress at mid-depth are 6.0 N/mm^2 and 4.3 N/mm^2, respectively. However, in regard to the actual shear fracture planes, strength values related to mid-depth constitute overestimations. An evalua-

Fig. 6 Typical shear failure within bond line interface
Abb. 6 Typischer Schubbruch im Klebfugeninterface

Fig. 7 Typical shear failure within a beech lamella
Abb. 7 Typischer Schubbruch innerhalb einer Buchenlamelle

Table 2 Results of the beam shear strength tests on structural sized beech GLT
Tab. 2 Ergebnisse der Schubfestigkeitsprüfungen an Buchen-BSH-Biegeprüfkörpern in Bauteilgröße

	Beam type and beam no.	Failure mode	Failure type	Failure location		Shear force capacity $V_u = F_u/2$ kN	Density ρ_{12} kg/m^3	Shear strength f_v				Experimental beam stiffness S kN/mm
				Lam. position	Distance from $z = 0$ mm			EBT		FEM		
								$f_{v,0}$ N/mm^2	$f_{v,f}$ N/mm^2	$f_{v,0}$ N/mm^2	$f_{v,f}$ N/mm^2	
WRH—with red heartwood	1	S	Bond line	−5/−6	190	187.8	698	5.44	3.89	5.35	4.03	22.13
	2	S	Bond line	−5/−6	190	190.8	683	5.52	3.95	5.43	4.09	23.63
	3	S	Wood	−1/−2	38	236.1	691	6.84	6.76	6.72	6.67	21.98
	4	S	Wood	+5	−190	257.2	696	7.45	5.33	7.33	5.52	22.51
	5	S	Bond line	−3/−4	114	161.4	696	4.67	4.19	4.60	4.20	21.61
	6	S	Wood	+1/−1	0	257.6	680	7.46	7.46	7.34	7.34	23.47
	13	S	Wood	−3/−4	114	267.9	701	7.76	6.96	7.63	6.97	22.08
	Mean WRH	–	–	–	–	222.7	692	6.45	5.51	6.34	5.54	22.49
	CV [%]	–	–	–	–	18.9	1.2	18.9	28.0	18.9	26.2	3.4
	5-perc. $f_{v,k}$[a]	–	–	–	–	162.9	680	4.71	3.43	4.64	3.58	–
NRH—no red heartwood	7	S	Wood	+1/+2	−38	176.6	698	5.11	5.06	5.03	4.98	21.60
	8	S	Bond line	+1/−1	0	213.2	696	6.17	6.17	6.07	6.07	22.89
	9	B	Wood	−8	304	221.1	696	6.40	6.40	6.30	6.30	22.93
	10	S	Wood	−3/−4	114	181.8	698	5.26	4.72	5.18	4.73	21.98
	11	S	Bond line	−5/−6	190	234.2	688	6.78	4.85	6.67	5.03	24.40
	12	S	Bond line	−4/−5	152	208.7	669	6.04	4.94	5.94	4.99	23.39
	14	S	Wood	−3/−4	114	133.3	676	3.86	3.46	3.80	3.47	22.63
	Mean NRH	–	–	–	–	195.6	689	5.66	5.09	5.57	5.08	22.83
	CV [%]	–	–	–	–	17.5	1.7	17.5	19.3	17.5	18.3	4.0
	5-perc. $f_{v,k}$[a]	–	–	–	–	144.3	671	4.15	3.66	4.11	3.72	–
All	Mean	–	–	–	–	209.1	690	6.05	5.30	5.96	5.31	22.66
	CV [%]	–	–	–	–	18.9	1.4	18.9	23.8	18.9	22.6	3.7
	5-perc. $f_{v,k}$[a]	–	–	–	–	150.1	675	4.34	3.54	4.29	3.62	–

Symbols and abbreviations: [a]5-percentile based on lognormal distribution; failure mode: S = shear, B = bending; V_u = shear force capacity, F_u = ultimate load at failure; $f_{v,0}$ = shear strength at mid-depth of the beam, $f_{v,f}$ = shear strength at failure location, S = experimental beam stiffness; EBT = elementary beam theory; FEM = finite element method

tion of shear strength related to the depth locations $z = f$ of the actual fracture planes leads partly to considerably lower strength values and a higher resulting scatter affecting the fifth percentile determination. Mean and fifth percentile of shear strength related to the fracture planes, computed by elementary beam theory, are:

$$f_{v,f,\mathrm{mean}} = 5.3\ \mathrm{N/mm^2} \quad \text{and} \quad f_{v,f,05} = 3.5\ \mathrm{N/mm^2}.$$

These strength values are 12% and 18% lower as the corresponding results evaluated for mid-depth. The derived fifth percentile of shear strength $f_v = f_{v,f}$ conforms closely to the characteristic shear strength value of $f_{v,k} = 3.4$ N/mm^2 specified in the German technical building approval Z-9.1-679 for beech glulam (DIBt 2009). However, it should be mentioned, that the specified shear strength results were obtained in laboratory conditions with webs that contained almost no initial cracks. The recognition of shrinkage cracks accumulating during service life, thereby reducing the gross width of the beam, is presently addressed by either an implicit reduction of the virgin material strength (DIN 1052 2008) or by application of a crack factor EN 1995-1-1/A1 (2004/2008). Regarding the intended use of the specific hardwood glulam exclusively in service class 1, for the time being, a minor reduction of about 15% applied to beam width or shear strength seems sufficient. In this sense, a crack factor of $k_{\mathrm{cr}} = 0.85$ or a design relevant characteristic shear strength of 3 N/mm^2 would be obtained.

The measured global beam deflections and hereof derived stiffnesses S showed very low scatter and no significant difference between the WRH and NRH sub-samples. For the combined sample WRH + NRH, a CV of 4% was obtained. The mean experimental stiffness of all beams evaluated in the load range of 0.1–0.4F_u was 22.7 kN/mm, being 12% lower as compared to the FE-computed value specified in Sect. 3.1.2. The obtained agreement is still satisfactory; nevertheless, the reasons for the stated discrepancy will be investigated further.

4.2 Block shear strength tests

Table 3 gives a compilation of the block shear strength results of the individual beams, separately for the WRH and NRH sub-samples and for the combined sample WRH + NRH. The mean block shear strengths of the individual sticks spanned from 11.3 N/mm^2 to 14.6 N/mm^2. The results of the two sub-samples WRH and NRH revealed no statistically significant difference, neither for shear strength nor for **w**ood **f**ailure **p**ercentages (WFP). For the combined sample (204 glue lines) the mean and fifth percentile values of block shear strength were $f_{v,\mathrm{BS}} = 12.8 \pm 2.3$ N/mm^2

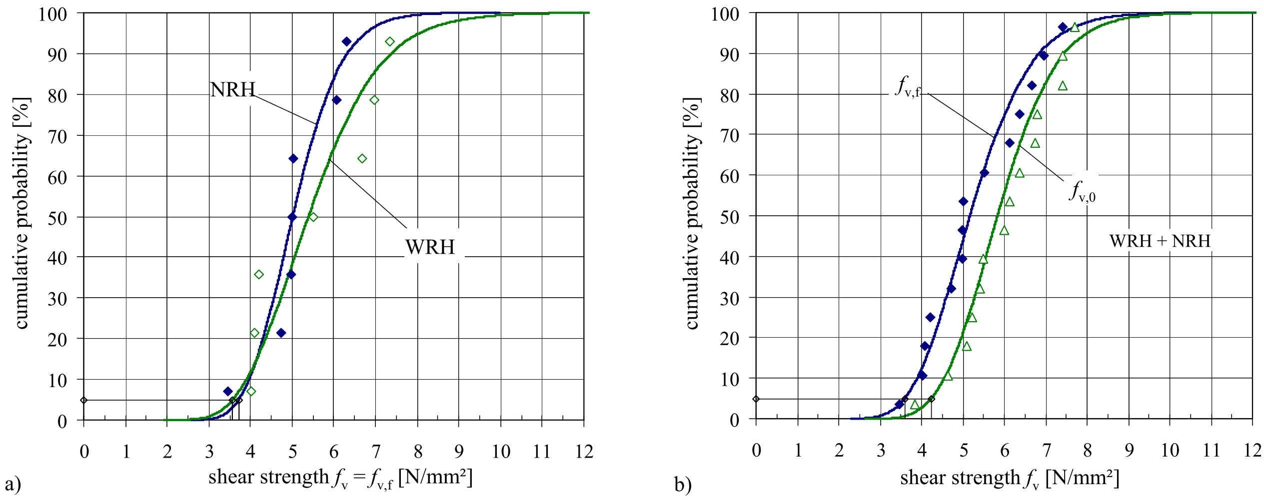


Fig. 8 Empiric cumulative frequency and fitted lognormal distribution of shear strength f_v evaluated by means of FE analysis (**a**) f_v at individual fracture plane location $z = f$ separately for both sub-samples WRH (◇) and NRH (◆); (**b**) f_v at mid-depth $z = 0$ (△) and at individual fracture plane location $z = f$ (◆) for the combined sample (WRH + NRH)

Abb. 8 Empirische kumulative Häufigkeit und angepasste Lognormal-Verteilung der Schubfestigkeit f_v gemäß FE-Berechung (**a**) f_v für die jeweilige Bruchebene $z = f$ getrennt nach den beiden Trägerkollektiven WRH (◇) und NRH (◆); (**b**) f_v in halber Steghöhe $z = 0$ (△) und für die jeweilige Bruchebene $z = f$ (◆) für das zusammengefasste Versuchskollektiv (WRH + NRH)

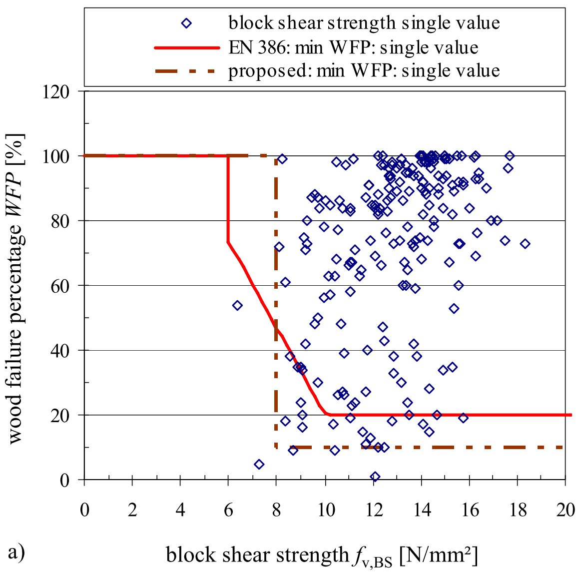
Table 3 Results of block shear tests acc. to EN 392 (1995)
Tab. 3 Ergebnisse der Blockscherversuche nach EN 392 (1995)

	Beam type and beam no.	n[a]	Block shear strength $f_{v,BS}$ Mean N/mm²	±SD N/mm²	CV %	min N/mm²	WFP[b] [%]
WRH—with red heartwood	1	14	13.6	2.6	18.8	9.1	73
	2	15	12.8	2.8	21.7	8.3	77
	3	15	13.2	1.8	13.4	10.5	72
	4	15	11.8	2.2	19.0	8.7	71
	5	14	11.8	2.3	19.5	6.3	75
	6	14	12.5	2.5	19.7	7.2	80
	13	15	12.7	2.3	18.4	8.9	76
	WRH	102	12.6	2.4	18.8	6.3	75
NRH—no red heartwood	7	13	13.1	1.8	13.9	9.2	74
	8	14	12.6	2.3	18.7	8.1	65
	9	15	13.6	1.9	13.8	9.3	74
	10	15	14.6	1.7	11.8	11.5	77
	11	15	14.2	1.8	12.5	9.8	78
	12	15	11.6	2.0	17.2	9.1	53
	14	15	11.3	1.9	16.8	8.3	48
	NRH	102	13.0	2.2	17.0	8.1	67
All	Mean	204	12.8	2.3	17.9	6.3	71

[a]Number of glue lines

[b]Wood failure percentage

and $f_{v,BS,05} = 9.3$ N/mm², respectively. The mean WFP for the combined sample WRH + NRH was 71%. The block shear results, revealing no significant difference between the WRH and NRH sub-samples, fully confirm the beam tests reported above, as well as earlier investigations on the effect of red heartwood on block shear strength of beech glulam (Frühwald et al. 2003; Aicher and Reinhardt 2007; Ohnesorge et al. 2006).



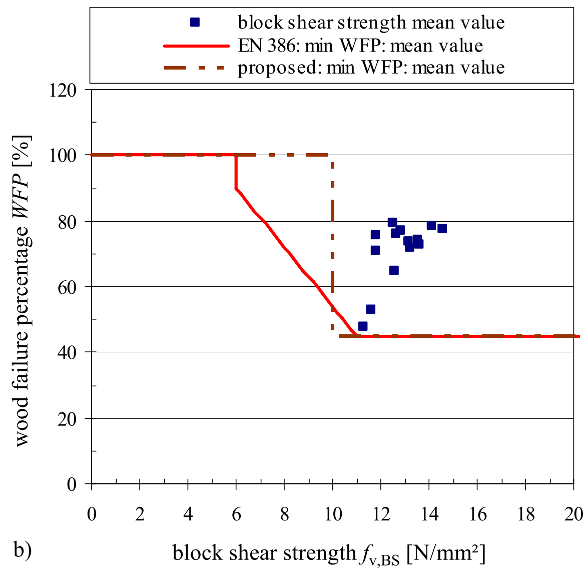


Fig. 9 Block shear strength and wood failure percentage of specimens from the tested beech glulam beams—current and proposed (*dotted line*) minimum requirements for production control (**a**) for single block shear strength values; (**b**) for mean block shear strength values
Abb. 9 Blockscherfestigkeit und Holzfaserbruchanteil von Proben aus den geprüften Buchen-BSH-Trägern—bisherige und empfohlene (gestrichelte Line) Anforderungen an den Mindestfaserbruchanteil (**a**) für Einzelwerte der Blockscherfestigkeit; (**b**) für Durchschnittswerte der Blockscherfestigkeit

Figure 9a and b depict the results of block shear strength and wood failure percentage of all individual specimens and the means of the 14 beams. No correlation between $f_{v,\mathrm{BS}}$ and WFP could be observed. The graph also shows the requirements (full line) for assumed $f_{v,\mathrm{BS}}$ − WFP relationships, as specified presently in the European standard EN 386 (2001). It is evident that the strength results fulfilled the requirements in almost all cases. Contrary the combined $f_{v,\mathrm{BS}}$ − WFP requirement was not met in about 10% of the individual results, whereas it was consistently met in the case of mean values.

A comparison of the block shear results with the requirements given in EN 386 (2001), there specified irrespective of a specific hardwood species, indicates that the present requirements on shear strength are set too low. On the contrary, the requirements on wood failure percentage appear to be rather rigorous. Figure 9a and b show conceivable proposals for revised requirements (dotted lines), which nevertheless have to be validated/improved by increased specimen numbers from varying product sources.

5 Conclusion

The shear force capacities of the investigated glulam beams of strength class GL 42c revealed no statistically significant influence of the red heartwood content in the bulk timber material and in the glued interfaces. Based on the depth locations of the actual fracture planes in the web, the lower lognormal fifth percentile of shear strength resulted in 3.5 N/mm^2. This value conforms very closely to the characteristic shear strength of 3.4 N/mm^2 given in the German technical approval Z-9.1-679 for beech glulam.

The block shear tests, similar to the beam tests, showed the red heartwood content of the glued laminations as having no effect, neither on shear strength nor on wood failure percentage. A comparison of the block shear results with EN 386 (2001) indicates that the present requirements on block shear strength in the European standard are not applicable to beech glulam. This should be confirmed by further investigations.

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